GEOTECHNICAL DESIGN FILE REPORT
SR 539 WIDENING PROJECT
TEN MILE ROAD TO THE
INTERNATIONAL BOUNDARY
WHATCOM COUNTY, WASHINGTON

HWA Project No. 96195

January 18, 1999

Prepared for:

PARSONS BRINCKERHOFF

HWA GEOSCIENCES INC.

January 18, 1999
HWA Project No. 96195

Parsons Brinckerhoff
999 Third Avenue, Suite 2200
Seattle, Washington 98104

Attention: Mr. Keith Nakano, P.E.

Subject: **GEOTECHNICAL DESIGN FILE REPORT**
SR 539 Widening Project
Ten Mile Road to the International Boundary
Whatcom County, Washington

Dear Keith:

In accordance with your request, HWA GeoSciences Inc. performed geotechnical studies for the SR 539 widening project in Whatcom County, Washington. The design file study reported herein included field explorations, laboratory testing, and preliminary engineering analyses for the proposed roadway widening project. Previously, HWA completed a draft design file report dated July 10, 1998 for your review. The following report incorporates your review comments of the draft report.

In conjunction with this report, HWA prepared an Environmental Assessment (EA) report dated June 8, 1998, which addresses geologic hazards and impacts for the SR 539 widening project. The accompanying report addresses geotechnical design file issues pertaining to the EA study.

We appreciate the opportunity of providing geotechnical engineering services on this project. Should you have any questions, or if we may be of further service, please do not hesitate to call.

Sincerely,

HWA GEOSCIENCES INC.

David L. Sowers, P.E.
Geotechnical Engineer

Ralph N. Boirum, P.E.
Principal Geotechnical Engineer

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1.0 INTRODUCTION

1.1 GENERAL

HWA GeoSciences Inc. (HWA) completed a geotechnical study in support of the proposed improvements to State Route (SR) 539 in Whatcom County, Washington. The project location is shown on the Vicinity Map, Figure 1. The purpose of this study is to explore and evaluate subsurface conditions along the project alignment, and based on the conditions encountered, provide preliminary geotechnical recommendations for the proposed improvements. A companion report prepared by HWA (June 8, 1998) addresses geologic hazards and impacts for the SR 539 Environmental Assessment (EA) study. This report addresses design file issues pertaining to the EA study.

1.2 PROJECT DESCRIPTION

SR 539 (the Guide Meridian Highway) extends due north from Interstate 5 in the City of Bellingham, next to Bellis Fair Mall, to Canadian Highway 13 at the Canadian border (Figure 1). The 24.4 kilometer (15.16 mile) highway is located in Whatcom County. It passes through the City of Lynden and intersects SR 544 and SR 546. The Washington State Department of Transportation (WSDOT), in cooperation with the Federal Highway Administration (FHWA), is proposing to widen SR 539 between Ten Mile Road and the Canadian border, a distance of 14.75 kilometers (9.16 miles). This section of SR 539 is currently a two-lane highway, with a three-lane section between Birch Bay-Lynden Road and Front Street within the City of Lynden.

Under the proposed action, the section of the roadway between Fishtrap Creek and the northern limits of the City of Lynden would be widened to a five-lane undivided highway with a two-way left-turn lane. It would consist of two 3.3-meter- (11-foot-) wide through lanes in each direction, a 3.6-meter- (12-foot-) wide two-way left-turn lane, as well as a 1.5-meter- (5-foot-) wide bike lane, and a 1.8-meter- (6-foot-) wide sidewalk and gutter on both sides of the highway.

The project segments south of Fishtrap Creek and north of the Lynden city limits would be widened to a four-lane access control highway with a restricted median. These segments would consist of two 3.6-meter- (12-foot-) wide through lanes in each direction with 3-meter- (10-foot-) wide shoulders. A median would separate the northbound and southbound lanes, except at intersections where left-left turn lanes would be constructed. SR 539 would be widened symmetrically, or in certain areas, to one side or the other to minimize potential impacts on adjacent land uses. A new parallel bridge across the Nooksack River would be constructed to accommodate the additional two lanes of southbound traffic. The existing bridge at Wiser Lake may be widened or a new parallel bridge may be constructed. The project also would include improvements to River Road, north of the Nooksack River, where the existing River Road would be extended parallel to SR 539 and a new intersection would be provided approximately 165 meters (560 feet) north of the existing intersection.

1.3 AUTHORIZATION AND SCOPE OF WORK

A proposal for this geotechnical study was submitted by HWA to Parsons Brinckerhoff on October 6, 1996. A subconsultant agreement was executed by Mr. Richard S. Page with Parsons Brinckerhoff on November 7, 1996. The scope of work completed for this project was consistent with that described in our proposal and included reviewing available geologic and geotechnical information, performing a subsurface investigation consisting of exploratory borings and hand auger holes, performing engineering analysis, and providing preliminary geotechnical recommendations for design and construction of the proposed roadway and bridge improvements.

2.0 FIELD AND LABORATORY INVESTIGATION

2.1 FIELD EXPLORATION

From May 15 through May 21, 1997, HWA monitored drilling of nine exploratory borings (designated BH-1 through BH-9) and 18 hand borings (designated HA-1 through HA-18) along the project alignment. Exploration locations are shown on the Site and Exploration Plan, Figures 2 through 6, and are described on the exploration logs in Appendix A. The locations of the explorations should be considered approximate. The exploratory borings were drilled to depths ranging from 8.8 meters (29 feet) to 28 meters (92 feet) using a truck-mounted drill rig under subcontract to HWA. The hand borings were advanced to depths ranging from 0.6 meters (2 feet) to 3 meters (10 feet) below the existing ground surface by HWA using hand equipment.

Soil samples were obtained at select intervals and taken to our laboratory for further examination and physical testing. Field exploration methods are described in further detail and logs of the explorations are presented in Appendix A.

2.2 LABORATORY TESTING

Laboratory tests were conducted on selected soil samples to characterize certain engineering and index properties of the soils encountered along the project alignment. Laboratory tests were conducted in general accordance with appropriate American Society for Testing and Materials (ASTM) methods for moisture content determination, grain size distribution, and Atterberg Limits. Moisture content test results are presented on the appropriate exploration logs in Appendix A. Grain size distribution and Atterberg Limits test results are presented in Appendix B.

2.3 PREVIOUS INVESTIGATIONS

Along the project alignment, numerous geotechnical reports and construction drawings have been prepared for various projects and are archived in the WSDOT libraries. The archived information includes boring logs, roadway design and resurfacing reports, and as-built drawings for bridge structures. The archived information has been prepared by either WSDOT or by private consultants. A summary of the available subsurface data from these previous studies is included in Appendix C.

3.0 GEOLOGIC AND SUBSURFACE CONDITIONS

3.1 REGIONAL GEOLOGY

The SR 539 Widening Project alignment is located in the northern portion of the Puget Lowland, part of a large glacial drift plain developed by repeated glaciations during the past 1.5 million years (Pleistocene Epoch). The southern Cordilleran ice sheet has occupied the Puget Lowland at least six times. Of these, the maximum advance of Cordilleran ice occurred during the Vashon stade of the Fraser Glaciation, approximately 15,000 years ago. Subsequent to the retreat of the Vashon stade, a period of glaciomarine deposition occurred during the Everson interstade. The last glacial advance occurred during the Sumas stade approximately 11,000 years ago. The Sumas stade is considered a minor oscillation in the overall retreat of the Cordilleran ice sheet (Easterbrook, 1992).

According to geologic mapping by Easterbrook (1976), the SR 539 project alignment traverses surficial deposits of alluvium, peat, and glacial outwash, as shown on Figures 7 and 8. Locally, fill material placed during highway construction is also present along the project alignment. Beneath the identified surficial units lies an older deposit termed Bellingham Drift. Each of these geologic units are described below in stratigraphic sequence from youngest to oldest.

- **Fill -** Embankment fill placed during highway construction is the most recent soil unit. These materials generally consist of loose to medium dense, well graded to poorly graded gravel with varying silt, sand and cobble content. Where encountered, the fill extended to depths ranging from less than 0.5 meters (1.6 feet) to about to 5.5 meters (18 feet). The deeper fills were encountered along the Wiser Lake and Nooksack River valley corridors.

- **Alluvium** – Alluvial (stream deposited) soils exist within the floodplain of the Nooksack River, and typically consist of stratified deposits of clay, silt, sand and gravel. The silty and clayey soils generally have high compressibility and low shear strength characteristics. Alluvium was observed in explorations BH-3, BH-4, BH-5, BH-6, HA-8, and HA-9. Based on our explorations, the alluvium extends from a minimum depth of about 0.3 meters (1 foot) to a maximum depth of 12.5 meters (41 feet). In some areas the unit may be interbedded with organic silt or peat.

- **Peat** - Peat deposits are mapped by Easterbrook (1976) in the vicinity of Wiser Lake and near the international boundary. Subsurface explorations by HWA indicate peat is present in the Nooksack River valley; peat was not encountered in explorations at Wiser Lake or near the Canadian border. It is possible that peat may have existed within these areas and was removed from beneath the roadway embankment prior to fill placement. Peat is a highly organic deposit which typically forms in depressions and channels, is generally dark brown to black, very soft, and has very high compressibility characteristics. Peat was observed at varying depths in explorations BH-3, BH-4, BH-6, and HA-18, with thicknesses ranging from 0.15 meters (0.5 feet) to 2.5 meters (8.2 feet).

- **Glacial Outwash** - Glacial outwash, deposited during the recession of the Sumas stade, is present along the majority of the project alignment. Glacial outwash consists of medium dense to dense, well sorted, stratified deposits of sand and gravel. Easterbrook indicates that the outwash grades more cobbly and gravelly towards the north end of the alignment. Glacial outwash was observed during our field investigation in hand auger holes HA-1 through HA-7 and HA-10 through HA-18. Outwash was observed in borings BH-1, BH-2, and BH-5 through BH-9, at depths of 2.2 meters (7.4 feet) to 27.1 meters (89 feet) below the ground surface. The glacial outwash extended to the maximum depth of exploration, or was underlain by Bellingham Drift deposits.

- **Bellingham Drift** - Bellingham Drift was deposited during the Everson interstade in a glacial marine environment. Bellingham Drift is described as medium stiff to hard, pebbly, unsorted, stratified, silt and clay. Where encountered (BH-2, BH-3, BH-4, BH-6), the top of this unit ranged from approximately 10 to 18 meters (33 to 59 feet) below ground surface along the project alignment. Bellingham Drift generally has moderate compressibility and low permeability.

3.2 SUBSURFACE CONDITIONS

The following sections summarize the subsurface conditions encountered within delineated segments of the project alignment.

3.2.1 Ten Mile Road to South of Wiser Lake

From Ten Mile Road to Wiser Lake the project alignment appears generally underlain by glacial outwash, as indicated by HWA hand hole logs (HA-1 through HA-6). Explorations performed at the Pole Road intersection by WSDOT and others indicate similar conditions. In the HWA hand holes, the glacial outwash encountered consists of loose to medium dense, fine to medium sand with varying silt and gravel content. Groundwater is relatively shallow; hand hole explorations in this section of the project indicated groundwater levels ranging from approximately 0.15 to 0.9 meters (0.5 to 3 feet) below existing ground surface at the time of exploration.

WSDOT (1978) hand auger explorations at Pole Road indicate that near-surface soils generally consist of 0.3 meters (1 foot) of gravel borrow over native sand and silty sand (interpreted to be glacial outwash). The explorations extended to a depth of 0.9 meters (3 feet). Two borings were later performed (WSDOT, 1987) at Pole Road and indicated medium dense, silty and gravelly sands underlie the site. These borings extended to maximums depths of 2.9 to 3.7 meters (9.5 to 12 feet). Other explorations were performed at Pole Road by Rittenhouse-Zeman & Associates (1989). Monitoring well logs and boring logs indicate the property on the northwest side of the intersection (Chevron gas station) is generally underlain by medium dense, sand and silty sand (interpreted to be glacial outwash). These explorations extended from 4.3 to 6 meters (14 to 20 feet) deep, with groundwater observed in some holes at approximately 4.3 meters (14 feet) below the ground surface.

3.2.2 Wiser Lake Causeway and Wiser Lake Bridge

Based on our borings BH-1 and BH-2, the causeway traversing Wiser Lake is composed of loose to medium dense gravel and sand fill. The thickness of the causeway embankment at the boring locations is approximately 5 meters (16.4 feet). We anticipate that the groundwater levels within the embankment will be approximately equal to the water level of Wiser Lake.

Although geologic mapping and as-built drawings indicate that peat may underlie the causeway embankment and vicinity, peat was not encountered in borings BH-1 or BH-2. It is possible that peat was removed from beneath the roadway embankment prior to fill placement. Glacial outwash was encountered underlying the embankment fill and extending to a depth of approximately 12.5 to 14.5 meters (41 to 47.6 feet) below ground surface. Bellingham drift underlies the glacial outwash and extended to the full depth of exploration.

WSDOT (1946) construction documents indicate that within the bridge footprint the top of the peat layer exists at about 3 meters (10 feet) below the existing bridge deck and is approximately 1.5 meters (5 feet) thick. The peat is underlain by fine sand (interpreted to be glacial outwash) to depths of between 5.2 to 8.8 meters (17 and 29 feet) below the bridge deck. Blue clay was observed below the fine sand.

3.2.3 North of Wiser Lake Bridge to East 74th Lane

Glacial outwash was observed in HWA hand hole HA-7 underlying undocumented fill. Hand hole HA-7 indicates that approximately 0.6 meters (2 feet) of fill overlies glacial outwash, which extended the full depth of exploration, 1.7 meters (5.6 feet). This subsurface information collaborates the geologic mapping which indicates this area is underlain by outwash.

WSDOT (1978) hand auger explorations at Wiser Lake Road indicate that near-surface soils generally consist of 0.3 meters (1 foot) of gravel borrow over native, medium dense, sand and silty sand (interpreted to be glacial outwash). The WSDOT explorations extended to a depth of 0.9 meters (3 feet).

3.2.4 East 74th Lane to Fishtrap Creek Bridge

HWA explorations indicate that this segment of the project alignment is underlain by alluvial deposits of variable thickness, as observed in borings BH-3 through BH-6 and hand holes HA-8 and HA-9. Road and bridge abutment fill were encountered at the ground surface, ranging in thickness from approximately 0.3 to 5.5 meters (1 to 18 feet) in the explorations. Underlying the fill, alluvium was observed ranging from 0.3 to 12.5 meters (1 to 41 feet) below the ground surface, and generally consisting of medium dense to dense sand with varying amounts of silt, to medium stiff to very stiff silt with sand. Isolated interbeds of soft, organic rich clay and peat are also present in this geologic unit. Bellingham drift and glacial outwash underlie the alluvium.

At the Nooksack River bridge, WSDOT (1950) construction documents indicate that the bridge footprint is underlain by 0 to 4.3 meters (0 to 14 feet) of fill, overlying medium dense to dense silt, fine sand, and fine gravel (interpreted to be alluvium). Explorations indicate that very dense/hard sand and blue clay exist below the alluvium to depths of 61 meters (200 feet) or greater below the ground surface. Subsurface information from WSDOT as-builts was not available for the south and north overflow bridges.

3.2.5 Fishtrap Creek Bridge to International Border

North of Fishtrap Creek to the border, the project alignment is underlain primarily by glacial outwash, as observed in explorations BH-7 through BH-9, and HA-10 through HA-18. Roadway embankment fill extends up to 3.8 meters (12.3 feet) deep and overlies the glacial outwash. The deepest fills were observed near the International border where geologic mapping indicates the presence of peat deposits. Prior to constructing the road embankment these peaty soils may have been overexcavated and replaced with roadway embankment fill. A layer of peat approximately 15 cm (6 inches) thick was observed in HWA hand hole HA-18.

Several WSDOT reports for facilities along the northern portion of the alignment indicate that the project alignment is underlain by fill and ditch sediments (depending on the off-set from roadway centerline), overlying medium dense to dense sand, silty sand, and gravelly sand deposits. The sand deposits are interpreted to be the glacial outwash unit and extended to the maximum depth of all explorations.

3.3 GROUNDWATER

Groundwater was encountered in 25 of our 27 explorations, at depths ranging from about 0.15 to 4.7 meters (0.5 to 15.5 feet) below the ground surface. The groundwater levels observed at each exploration are shown on the individual logs in Appendix A. It should be noted that observations of groundwater levels during drilling can be misleading, as the actual groundwater level is often somewhat higher than observed in a boring. Consequently, the localized water table may actually be higher than indicated during the exploration. The groundwater conditions reported are for the specific date and locations indicated, and therefore may not necessarily be indicative of other times and/or locations. Furthermore, groundwater conditions will vary depending on the season, local subsurface conditions, and other factors.

4.0 CONCLUSIONS AND PRELIMINARY RECOMMENDATIONS

4.1 GENERAL

Conclusions and preliminary recommendations pertaining to seismic hazards, soil liquefaction potential, bridge foundation types and sizes, feasible retaining wall types and applications, general embankment construction considerations, sheet pile walls, drainage and erosion considerations are addressed in the following sections of this report. Pavement design is not included in the scope of this study. Because the recommendations presented herein are preliminary and conceptual in nature, additional exploration and engineering analyses will be needed as part of the final design process.

The project site lies within Seismic Zone 3, as designated by the Uniform Building Code (ICBO, 1997). Consequently, moderate levels of earthquake shaking are anticipated to occur in the project area. At Wiser Lake, the Nooksack River, and Fishtrap Creek, new bridge crossings will be constructed across areas underlain by potentially liquefiable soils. New bridges and/or widened bridges at these locations should be supported on pile foundations to maintain support if the underlying soils liquefy. In our pile analysis, we assumed that during and following periods of earthquake shaking, piles would be supported exclusively by the non-liquefaction susceptible soils. We recommend piles consist of pre-cast concrete or steel pipe piles driven to depths sufficient to generate skin friction and end bearing resistance to support the required static and dynamic loads.

Along the Wiser Lake causeway, roadway widening is anticipated to extend symmetrically on both sides of the existing embankment. To minimize and prevent placement of fill material in the lake, retaining structures will be necessary to support the widened embankment. One proposed alternative involves using rows of steel sheet piling on each side of the causeway to support the new embankments. Steel sheet piling is typically produced in 12.2-meter (40-foot) lengths and can be driven using an impact or vibratory hammer.

Other retaining structures will be required at locations along the project alignment to support cut and fill slopes. Retaining wall options discussed herein include MSE (mechanically stabilized earth) retaining walls, soldier pile walls, and cantilevered concrete walls. Fill slopes can either be sloped at 2H:1V (horizontal:vertical) or flatter, or can be supported by retaining walls.

4.2 Seismic Hazards Evaluation

4.2.1 General

Seismic hazard areas are generally defined as areas subject to severe risk of earthquake damage as a result of seismically induced settlement or soil liquefaction. Since the 1850’s, at least 25 earthquakes of Magnitude 5.0 (Richter Scale) or greater have reportedly occurred in the Puget Sound and North Cascades region. Four events may have exceeded Magnitude 6.0. These include a 1949 event near Olympia (Magnitude 7.1), and a 1965 event centered between Seattle and Tacoma (Magnitude 6.5). The subduction of the Juan De Fuca plate beneath the North American plate is believed to directly or indirectly cause most of the earthquakes in Washington (Noson et al., 1988).

The project area lies within Seismic Zone 3, which includes the Puget Sound region, and represents an area of significant seismic risk. For comparison, much of California and southern Alaska are in Seismic Zone 4, which is one of higher seismic risk.

Consequently, moderate levels of earthquake shaking may be anticipated during the design life of the subject facilities.

Design ground acceleration for the project alignment was determined using results from the seismic maps presented in the Bridge Design Manual (WSDOT, 1998), based on the *National Seismic Hazards Mapping Project* completed by the USGS in 1996. WSDOT's maps present horizontal bedrock accelerations associated with a 10 percent probability of exceedance in a 50-year period. Interpolation between isoseismic contours on the WSDOT design maps indicate a design peak ground acceleration (PGA) along the project alignment is approximately 0.22g. For comparison, the American Association of State Highway and Transportation Officials (AASHTO) design seismic maps indicate the PGA is approximately 0.18g for the Whatcom County area (AASHTO, 1993). It is important to note, however, that the AASHTO maps have not been revised to incorporate the results of the recent USGS mapping and are outdated in our opinion.

Based on the UBC (ICBO, 1997) soil classification method, the glacial and alluvial soils found along the project alignment are classified as Soil Type S_D.

Potential earthquake effects may include soil liquefaction potential, seismic-induced settlement, or ground fault hazards. The following sections provide a general discussion of these seismic issues for use at the planning level; more detailed investigations and analysis will be required to address seismic issues at the design level.

4.2.2 Soil Liquefaction Potential

Liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore water pressures in saturated soils and subsequent loss of strength of the soil. In general, liquefaction-susceptible soils include loose to medium dense, clean to silty sands below the water table. Typically, soil liquefaction is more likely to occur in river and marine environments where the soils are saturated, poorly sorted, and have low relative densities. In glacial environments where soils have been over-ridden by ice, soil liquefaction potential is negligible.

The evaluation of liquefaction potential is complex and dependent on numerous site parameters including soil grain size, soil density, site geometry, static stresses, and the seismic ground acceleration. Typically, the liquefaction potential of a site is evaluated by comparing the cyclic shear stress ratio (the ratio of the cyclic shear stress to the initial overburden stress) induced by an earthquake to the cyclic stress ratio required to cause liquefaction. At the preliminary level, we evaluated the earthquake-induced cyclic shear stress ratio for the design earthquake using an empirical relationship developed by researchers for this purpose (Seed and Idriss, 1982). The design seismic event was assumed to be a 500-year earthquake with a magnitude of 7.5 (Richter scale) and a peak ground acceleration of 0.22 g.

For the Wiser Lake area, our preliminary liquefaction analysis indicates that approximately 4.5 meters (14.8 feet) of the loose to medium dense gravelly fill and medium dense sand deposits located approximately 3.5 to 8 meters (11.5 to 26.3 feet) below the ground surface will have moderate potential for liquefaction during the design earthquake. For the Nooksack River and floodplain area, available subsurface information (boring BH-5) suggests that approximately 2 meters (6.6 feet) of soil located at 4 meters depth has a moderate to high potential for liquefaction during the design earthquake. In general, other borings in the floodplain area did not encounter soils considered potentially liquefiable. A preliminary evaluation of other areas along the alignment indicated that soils underlying the project alignment have low to negligible soil liquefaction potential, i.e. most of the alignment is underlain by glacial outwash deposits which are not liquefiable. It should be noted that this liquefaction assessment is based on limited subsurface data. Additional liquefaction analyses should be performed during the design phase.

4.2.3 Seismic Induced Settlement

Settlement of the ground surface may occur as a result of earthquake shaking, particularly in conjunction with the occurrence of soil liquefaction. Vertical settlement of soil around piles could result in high downdrag loads, which could result in substantial damage if the piles do not have sufficient embedment below liquefiable soils. High lateral loads may also be exerted on pile foundations in situations where liquefiable soils cause lateral spreading. Any new foundation piles should be designed to resist loading related to seismic/liquefaction events.

4.2.4 Lateral Spreading

Lateral spreading involves lateral displacements of large volumes of liquefied soil. Lateral spreads frequently occur in river or marine environments when unsupported, saturated soil masses move toward a free face during periods of earthquake shaking. Based on our analyses of the Wiser Lake causeway embankment materials, some lateral spreading of the causeway may occur during an earthquake. Causeway widening should include measures to support existing new and fills, including sheet pile walls or other retaining wall types, to reduce lateral spreading potential.

4.2.5 Ground Fault Rupture Hazard

Our review of available literature (including Cheney, 1987; Crossen, 1972; Noson et al., 1988) did not indicate the potential presence of any active ground faults on or in the immediate vicinity of the project alignment. Also, during our site reconnaissance, we did not observe any evidence of active faulting or lineaments which might be indicative of recent surface faulting. Based on this evidence, we conclude that ground fault rupture hazard is low.

4.3 BRIDGE FOUNDATIONS

4.3.1 General

As-built drawings for each of the existing bridge crossings indicate the bridges are supported on friction piles embedded primarily in medium dense sands and medium stiff to stiff silts. The piles consist of either reinforced concrete piles or composite piles; composite piles are a combination of timber and reinforced concrete. The drawings indicate allowable capacities for the existing foundation piles are relatively low, ranging from 20 to 30 tons. Penetrations for existing piles range from about 14 to 95 feet. Generally, the existing piles either terminated in the medium dense sands above a depth of 30 to 35 feet, or they punched through the sand layers and penetrated considerably deeper, into the underlying silt and clay layers. These silt and clay layers are relatively soft and compressible and provide bearing support only for relatively lightly loaded piles.

Because they are supported on friction piles that bear in and above compressible soils, it is likely that the existing bridge structures have experienced several inches of settlement. The settlements do not appear to have been damaging. In our opinion, the existing structures are not likely to settle significantly further due to the present loading conditions. However, the existing structures will likely settle if large embankments are placed in close proximity to the existing bridge abutments. Borings at/near bridge crossings in the Nooksack River valley and along Wiser Lake encountered deposits of peat and compressible silt and clay below the existing fill embankments. New embankments placed within a distance equal to the depth to the underlying compressible soils (where present) will cause additional downdrag forces (negative friction forces) to act on the existing piles. We recommend placing embankments at a distance away from existing abutments that is greater than the depth to the bottom of the compressible soils so as not to cause additional load on the existing structures. (i.e. proposed bridge approaches will be longer than the existing approaches).

New piles of similar size and depth, and loaded similarly to the existing piles should also be expected to settle several inches. If several inches of differential settlement between proposed and existing structures are acceptable, the proposed structures could be supported on 20- to 30-ton (allowable capacity) friction-bearing piles of similar lengths as the existing piles. We recommend using pre-cast concrete piles where lower capacity friction piles are needed. For higher capacity end-bearing piles, we recommend using steel pipe piles. The steel pipe piles can be used where the design length of the piles will vary significantly and where splicing is necessary to achieve greater depths. Pre-cast concrete piles are not easily spliced and are typically installed where the required pile lengths are less than 100 feet. Further explorations should be performed prior to selecting final pile types and lengths.

The amount of differential settlement between new embankments/bridges and existing embankments and structures will depend primarily on the thickness of the peat layers. To avoid differential settlements between new bridge abutments and new embankments, the new structures could be supported on piles that bear in deeper, denser soils than the existing piles. Deeper soil borings will be needed to determine the necessary length for higher capacity piles, as the existing borings did not encounter glacially consolidated soils.

Final design of the proposed bridge foundations will also depend on the configuration of piling at the pier locations and pile spacings. Generally, end bearing piles will not require a reduction in capacity for group action, whereas, a reduction factor should be applied to groups of friction piles, particularly when supported by cohesive soils (silts and clays). The reduction factor for pile groups will depend on the number of piles in the group and the pile spacing.

4.3.2 Wiser Lake Bridge

Options at Wiser Lake for roadway and bridge widening include supporting the widened portion of the causeway and bridge on an entirely new bridge structure, or widening the causeway symmetrically using sheet pile walls and fill and widening the bridge structure. For preliminary design and planning purposes the information provided herein may also be used to conceptualize design requirements for both a widened bridge or a pile-supported causeway. Final foundation design will depend in part on the configuration of the new bridge and causeway.

The existing three-span bridge is supported by driven, octagonal, pre-cast reinforced concrete piles having a nominal diameter of 33 cm (13 in.). Each of the four bents supporting the bridge deck is comprised of seven concrete piles spaced at 1.8 meters (6 feet) center to center. The concrete piles have an average penetration of 4.1 to 5.2 meters (13.5 to 17 feet) and were originally driven to a bearing capacity of 270 kN (30 tons) each.

Soils encountered in borings BH-1 and BH-2 (located south and north of Wiser Lake Bridge, respectively) generally consist of about 4.8 meters (15.6 feet) of loose to medium dense sand and gravel fill, overlying medium dense to dense glacial outwash sand. Medium stiff clay and silt are present below the outwash to a depth of at least 22 meters (72 feet). Although not observed in BH-1 or BH-2, peat of limited thickness and extent is anticipated to be present overlying the glacial outwash, based on geologic mapping by Easterbrook (1976), and as-builts drawings for the Wiser Lake bridge.

Our preliminary analyses indicate that 36 cm (14 in.) nominal diameter octagonal pre-cast concrete piles driven to a tip elevation of about 14 meters, (about 5 meters below the existing ground surface, should be capable of supporting working loads of 270 kN (30 tons) per pile, similarly to the existing piles. In our opinion, the outwash and underlying medium stiff clays will not provide adequate support for driven piles of capacities greater than 270 kN (30 tons) without large settlements. If larger capacity piles are desired, we anticipate piles will have to be driven considerably deeper, potentially to depths beyond the limits of borings BH-1 and BH-2. High capacity piles (80 to 100 ton working loads) would have to bear in glacially consolidated soils. If large capacity piles are needed, two deep borings should be drilled to determine the depth to very dense, glacially consolidated soils that underlie the clay.

The performance (settlement) of the new/widened causeway and bridge will depend on whether existing peat and organic soils are removed or preloaded. If soft soils are removed, as they were during original construction, installation of piling and construction of the causeway should be able to proceed without settlement mitigation measures. If these soft soils are not removed (the anticipated option for environmental reasons), preloading will be necessary before pavements are placed and new bridge piling installed. Preloading will also minimize the effects of downdrag on new piles driven through the embankments. As discussed briefly above, existing piles will also be affected by new embankments placed within a distance equal to the depth to the base of the compressible soil. These embankments will exert downdrag forces to the existing piles. Based on the as-built drawings, compressible soils extend/extended to about 5 meters (16.4 feet) below the proposed widened portions of the new causeway. Consequently, new fills should not be constructed within 5 meters of the existing bridge piers or additional settlement of the bridge abutments could occur as the embankments are constructed.

If the final causeway design is exclusively supported on piles or pile groups, downdrag forces are not an issue. If the final causeway design includes embankments and pile supported bridges, we recommend removing the peaty soil or preloading so as to minimize the amount of settlement the embankment will experience.

4.3.3 South Overflow Bridge

The existing south overflow bridge is supported by 24 bents consisting of 8 driven composite piles per bent; the diameter of these piles was not available on the as-builts. The average penetration per bent ranges from approximately 25.3 to 29 meters (83 to 95 feet), with each pile jetted and driven to a minimum capacity of 200 kN (22 tons). In boring BH-3 (located near the south abutment) sand and silt fill was observed extending from the ground surface to a depth of approximately 4.8 meters (15.6 feet). Below the fill, alluvium consisting of soft to medium stiff organic silt and clay with interbeds of soft peat was observed to a depth of 8 meters (26 feet) below the ground surface. From 8 to 10 meters (26 to 33 feet) the alluvium transitioned to medium dense to dense, poorly graded sand with few gravels. Medium stiff to very stiff Bellingham Drift was encountered beneath the stratified alluvium and extended to the full depth of boring BH-3.

As currently envisioned, a parallel south overflow bridge will be constructed west of the existing bridge as part of roadway widening. Preliminary liquefaction analysis of soils in this vicinity (boring BH-3 located on the south approach of the south overflow bridge) indicates that liquefaction susceptibility is low. However, settlement of the soft peat will likely occur as the embankments are constructed and could add additional downdrag loads on the existing and proposed pile foundations. We recommend preloading the alignment prior to pile driving to minimize the downdrag forces on the piles, or design the piles (lengthen them) to withstand the additional downdrag load if the embankment settles.

Preliminary pile analyses indicate that 42 cm (16½ in.) octagonal concrete piles or 46 cm (18 in.) steel pipe piles driven into the medium stiff to stiff clay would need to be driven beyond the bottom of our boring (BH-3) to support working loads of 270 to 360 kN (30 to 40 tons). Smaller capacity piles (20 to 25 tons allowable capacity) of similar diameters could be driven to depths of 24.4 to 30 meters (80 to 100 feet), similar to the existing piles. If large capacity piles are needed, two deep borings should be drilled to determine the depth to very dense, glacially consolidated soils which underlie the clay.

To prevent additional downdrag forces from being added to the existing piles, we recommend against placing new embankment fills next to existing piles within a distance equal to the depth to the base of the compressible peat, organic silt and organic clay, or about 8 meters (26.2 feet). This will require extending the span of the new bridge approximately 8 meters on each end further than the existing bridge span.

4.3.4 Nooksack River Bridge

The existing Nooksack River bridge foundation is supported by 15 piers, with each pier supported by composite piles; the south approach is supported by five piers, two piers support the single span steel truss, and eight piers support the north approach. Each pier in the south approach is comprised of 11 piles, driven to average penetrations varying from 21.2 meters (69.8 feet) at Pier No. 1, to 11.5 meters (37.7 feet) at pier No. 5, with the piers numbered sequentially from south to north. Piers 6 and 7, supporting the steel truss bridge, are composed of two groups of driven piles per pier, with 56 piles per group.

The average penetration of the piles is 10.3 meters (33.9 feet) for Pier 6, and 21.3 meters (70 feet) for Pier 7. Pier 8, supporting a portion of the north approach, is comprised of one group of 48 piles, with average penetration of 25 meters (82.1 feet). Pile embedments were not available for Piers 9 through 15. The as-built drawings indicate that each pile was driven to a minimum capacity of 200 kN (22 tons). The diameter of the installed piles was not available on the as-built drawings.

Soil conditions observed near the south abutment of the Nooksack River Bridge in boring BH-4 are similar to those encountered in nearby boring BH-3; approximately 5.5 meters (18 feet) of medium dense to dense silty sand fill over about 7 meters (23 feet) of alluvium, composed of 4.5 meters (14.8 feet) of medium stiff to stiff silt with variable sand content, over about 2.5 meters (8.2 feet) of medium stiff peat. Underlying the peat to the full depth of our exploration is Bellingham Drift, which consists of medium stiff to stiff silt and lean clay with varying amounts of sand. Liquefaction analysis indicates that soils in the Nooksack River bridge vicinity have low liquefaction susceptibility.

Similar to the foundation for the south overflow bridge foundation, either pre-cast concrete or steel pipe piles could be used to support the Nooksack River bridge abutments. We anticipate that 42 cm (16½ in.) octagonal concrete piles or steel pipe piles 46 cm (18 in.) in diameter filled with concrete would need to be embedded to similar depths to the existing piles to support design loads of 180 to 225 kN (20 to 25 tons) per pile. Piles supporting loads of 270 to 360 kN (30 to 40 tons) would need to be founded beyond the depths of our explorations. The final pile depths will depend in part on whether the abutment fills are used as a preload before pile driving, as settlement of the soft peat could add additional downdrag loads to the new piles. Piles which are not driven through the embankment fill (e.g. bridge piers located near the center of the Nooksack river bridge) would not have to be designed to resist downdrag forces.

We recommend against placing new embankment fills next to existing piles within a distance of about 12.5 meters (41 feet); 12.5 meters is the distance equal to the depth to the base of the compressible peat. This will require extending the span of the new bridge approximately 12.5 meters on each end further than the existing bridge span.

4.3.5 North Overflow Bridge

Similar in design to the south overflow bridge, the existing north overflow bridge is supported by 17 bents consisting of 8 driven composite piles per bent; the diameter of these piles was not available on the as-builts. Average pile penetration ranges from about 14.2 to 18.2 meters (47 to 60 ft) and each pile has been driven to a minimum capacity of 200 kN (22 tons). Soil conditions encountered in boring BH-5 at the south abutment of the north overflow bridge consisted of about 3.2 meters (10.6 feet) of medium dense to dense sand fill over about 8.5 meters (28 feet) of alluvium, composed of medium dense to dense poorly graded sand with silt. Underlying the alluvium is medium dense to dense glacial outwash, extending to the full depth of exploration. Compressible peat or organic soils were not observed during drilling BH-5.

Preliminary liquefaction analysis indicates that the upper 1 to 2 meters (3.3 to 6.6 feet) of alluvium is susceptible to liquefaction during the design earthquake. Subsequent liquefaction-induced settlement could cause downdrag forces to develop in the overlying fill material. The effects of downdrag forces acting on the abutment and pile group were taken into account in the following analysis.

Preliminary pile analyses indicate that 36 cm (14 in.) nominal diameter octagonal pre-cast concrete piles or steel pipe piles which are 41 cm (16 in.) could support working loads of 270 (30 tons) per pile, provided they are embedded at least 12.1 to 15.1 meters (40 to 50 feet) below the ground surface, and into the glacial outwash sands. At this depth of embedment, the piles sizes presented herein should support downdrag affects acting on the piles if the upper alluvial soils liquefy.

Unlike other borings in the Nooksack River valley performed by HWA, BH-5 did not encounter peaty or organic soils. If during future explorations along the alignment compressible soils are encountered, the existing and proposed piles and new embankments should be designed to mitigate the potential differential settlement.

Based on subsurface conditions encountered in our other borings, new piles that extend deeper than 27.1 meters (89 feet) may encounter weaker Bellingham Drift soils. Consequently, we anticipate small capacity piles could be used at this location so that the shorter piles can be hung up in the denser outwash sand. If large capacity (and potentially longer) piles are needed, two deep borings should be drilled to determine the depth to very dense, glacially consolidated soils (anticipated to underlie the clay), where the piles could be supported.

4.3.6 Fishtrap Creek Bridge

The existing Fishtrap Creek bridge foundation is composed of 6 bents, with 8 composite piles per bent. Piles were driven to average penetrations ranging from 16.7 to 18.8 meters (55 to 62 feet) and to minimum capacities of 200 kN (22 tons) per pile. Boring BH-6 was drilled near the north abutment of the Fishtrap Creek bridge and indicated approximately 2 meters (6.6 feet) of loose silty sand with gravel fill was placed over 1½ meters (5 feet) of very soft peat and 3 meters (10 feet) of alluvium. Beneath the alluvium, approximately 11½ meters (38 feet) of glacial outwash was encountered. Bellingham drift underlies the glacial outwash, extending from 18 meters (59 feet) to at least 24.1 meters (79 feet), or the full depth of the boring.

As currently envisioned, a second bridge will be constructed west of the existing bridge as part of roadway widening. Preliminary liquefaction analysis based on subsurface data from boring BH-6 indicates that alluvium and glacially deposited soils at Fishtrap Creek have a low liquefaction potential. We anticipate working load capacities of 200 kN (25 tons) per pile can be obtained using 36 cm (14 in.) nominal diameter pre-cast octagonal concrete piles or 41 cm (16 in.) steel pipe piles filled with concrete, provided they are hung up in the medium dense sandy soils at 15 to 18 meters (50 to 60 feet). To support greater working loads, we anticipate piles would need to extend beyond the depth of boring BH-6. The bottom of this boring indicates that softer clay deposits that will provide little end bearing resistance for piles exist at depth. High capacity piles would need to extend through the clay to denser, glacially consolidated deposits. Additional borings will need to be performed at Fishtrap Creek bridge to determine the length of piling that would be required to reach the glacially consolidated deposits.

We recommend against placing new embankment fills next to existing piles within a distance equal to the depth to the base of the compressible peat, or about 3.5 meters (11.5 feet). This will require extending the span of the new bridge approximately 3.5 meters further on each end than the existing bridge span.

4.4 General Retaining Wall Considerations

4.4.1 General

Retaining walls are anticipated at various locations along the project alignment. Selection of specific retaining wall types will depend on numerous factors, including type of slope (cut or fill), height of slope, subsurface conditions, proximity of adjacent structures or traffic lanes, anticipated settlement, and aesthetic appearance. The following report sections present four retaining wall alternatives suitable for supporting the proposed cuts and fills, and include sheet pile walls, mechanically stabilized earth (MSE) walls, conventional concrete cantilevered walls, and soldier pile walls. Design and construction advantages and disadvantages are discussed herein.

4.4.2 Sheet Pile Walls

One alternative under consideration for roadway widening through the Wiser Lake corridor is to use sheet pile walls to support embankment fills. As widening is anticipated to be symmetrical along the alignment, a single row of steel sheet piling would be driven on each side of the roadway. Placement of sheet piling at the toe of the existing embankment is being considered because it will minimize disturbance to the lake environment during fill placement. Based on the height of the anticipated causeway embankment, we anticipate the cantilevered portion of the wall would be approximately 4.6 meters (15 feet) high.

Steel sheet piling is available in various gauge thicknesses and typically in 12.2 meter (40-foot) sections. Sheet piling is typically driven using an impact or vibratory hammer. Based on the subsurface conditions and proposed causeway dimensions, we recommend cantilevered sheet pile walls be embedded 1½ to 2 times the free-standing, exposed height of wall. Specific design parameters should be developed following more detailed geotechnical investigation for this alternative. Use of dead-man anchors or tie-backs may result in a more economical design for large wall heights.

As discussed previously, some soil liquefaction of subgrade soils may occur within this portion of the alignment. Soil liquefaction could potentially result in vertical displacements (settlement), lateral displacements or a combination of both. The use of steel sheet piling may help limit the amount of lateral displacement, or spreading which could occur during a seismic event. Not only will sheet piling help to support the new fill embankment, it will help support the existing causeway embankment which is susceptible to both vertical and lateral displacements if retaining structures are not part of the design solution.

Steel sheet piling used in a fresh-water environment will be susceptible to some corrosion, although less than if used in a salt-water environment. Protecting sheet piling is important to minimize rusting and deterioration of the steel wall. Protective measures may include, but are not limited to, placing rock and soil fill against the exposed face to act as a protective barrier, using coatings that protect the steel sheet piling, cathodic protection, and/or using thicker gauge steel sheeting to allow for some “sacrificial” corrosion to occur.

4.4.3 Mechanically Stabilized Earth (MSE) Walls

Mechanically stabilized earth retaining walls consist of alternating layers of backfill soil and reinforcing material with facing elements. Commonly used reinforcing elements include steel strips and various geosynthetic products such as geogrid and geotextile sheets. The vertical spacing of the reinforcing elements is typically on the order of 0.3 to 1.0 meter (1 to 3.3 feet), depending on the reinforcing material specified and other parameters. Commonly used facing elements include pre-cast concrete panels or blocks, gabion baskets, and wrapped-face geosynthetics.

MSE walls are considered feasible to support new embankments along the Wiser Lake causeway. However, because of the geogrid reinforcing layers, a large area behind the wall is required for construction. This type of wall is better suited for larger areas where work activities are not restricted. These constructability issues would result in additional costs if MSE walls are used for causeway widening. The additional room required for construction could also have impacts on the routing of traffic during construction. However, an MSE wall is an excellent alternative for fill slopes where right-of-way or other restrictions prevent placing large sloped embankments. MSE walls have a relatively low unit cost and have large tolerances to differential settlement.

Many MSE wall systems are available as proprietary wall systems. Three proprietary MSE wall systems (VSL, Reinforced Earth, and Hilfiker) have been pre-approved by WSDOT, as indicated in a memorandum prepared by the WSDOT Materials Laboratory (WSDOT, 1996). These wall systems are pre-approved for heights up to about 9 meters (30 feet), when slopes above the wall are 2H:1V or flatter. Because the new retaining walls along the alignment are anticipated to be less than 9 meters high, these pre-approved wall systems are applicable.

4.4.4 Concrete Cantilever Retaining Walls

Concrete cantilever retaining walls are readily installed without specialized equipment, and are usually economical to construct up to heights of roughly 3 to 5 meters (10 to 16.4 feet). The principal disadvantage of conventional concrete retaining walls is that these walls have relatively low tolerance of differential settlement. Conventional concrete cantilever retaining walls should only be used where large differential settlements are not expected, or where soft, compressible soils can be overexcavated and replaced with compacted structural fill. Concrete walls should be pile-supported for cases where compressible soils are not (or can not be) removed.

The lateral earth pressures against concrete retaining walls depend upon the inclination of the back-slope, degree of wall restraint, type of backfill, method of backfill placement, degree of backfill compaction, drainage provisions, and magnitude and location of any adjacent surcharge loads. Site-specific conditions and geotechnical parameters for each proposed wall location should be evaluated prior to design.

4.4.5 Soldier Pile Walls

Soldier pile walls typically consist of drilled in-place HP or wide flange steel sections on approximately 2- to 2½-meter (6- to 8-foot) centers. The solider piles should be embedded sufficiently to provide lateral support of the cut or fill soils. The annular space around drilled in-place piles is filled with structural concrete to the base of the wall and the portion of the hole above the planned base of the wall is backfilled with a lean concrete mix. Timber lagging can then be placed to span between the soldier piles to transmit the lateral earth pressure loads of the retained soils to the soldier piles. For aesthetic purposes, facing elements can be placed over the timber lagging.

One advantage of soldier pile walls used to support cuts is that they can be constructed in a relatively confined space. As described above, the soldier piles are typically installed before performing excavations for proposed cuts. Excavation and placement of timber lagging are typically performed in 1.2- to 1.8-meter (4- to 6-foot) increments, in a “top-down” sequence.

Lateral earth pressures against soldier pile walls will depend on the height of the wall, inclination of the back-slope behind the wall, type of material being retained, drainage provisions, magnitude and location of surcharge loads, and other factors. Site specific conditions and geotechnical parameters should be evaluated during the design phase.

4.5 SETTLEMENT CONSIDERATIONS

During our subsurface investigation, compressible soil layers (e.g. peat and organic clay) were encountered at various locations along the project alignment. As-built drawings for existing structures and available geologic references indicate peat and/or organic soils may still exist under portions of the alignment in areas not identified during this phase of investigation. Some evidence suggests compressible soils may have been removed for embankments at Wiser Lake and in the Nooksack River vicinity during subgrade preparation. Generally, though, our explorations indicate soft peat and organic clay layers still underlie portions of the existing embankment, and potentially these layers may extend beneath the footprint for the proposed roadway widening. Explorations which encountered peat include BH-3, BH-4, BH-6, and HA-18.

In some cases, compressible soils beneath existing embankments have consolidated under the weight of the fill over a period of many years and settlement is essentially complete. The addition of new embankment fill along the existing roadway, though, may result in additionally settlement if these soils are not removed before placing new fill.

Methods for mitigating settlement of new embankments involves using preloading or surcharging techniques. Preloading involves placing the proposed embankment fill soils, allowing the weight of the embankment to consolidate the underlying soft soils for a predetermined time limit, and then completing road construction and paving. Surcharging is a modification of the preloading process and involves placing additional fill material on the proposed embankment fill to accelerate the rate at which consolidation occurs. The additional fill or surcharge material is removed and used in other project areas or is hauled off-site. Both of these techniques are useful in achieving consolidation of soft soils. Additional investigations (field and laboratory), and engineering analysis of settlement potential will be required during final design.

4.6 GENERAL EARTHWORK CONSIDERATIONS

4.6.1 Embankment Fill Slopes

Embankments should be constructed in accordance with Section 2-03.3(14) of the 1998 WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction*. We recommend fill slopes be constructed at inclinations of 2H:1V or flatter. At these inclinations, slopes are anticipated to have adequate factors of safety when considering overall slope stability. Material and compaction requirements are also included in Section 2-03(14).

4.6.2 Permanent Cut Slopes

As described earlier, some roadway widening may require cuts into native slopes. Based on the soil conditions encountered in our explorations, we anticipate permanent cut slopes could be sloped at maximum inclinations of 2H:1V to 3H:1V. The actual slope inclination will depend on the local subsurface conditions in the vicinity of the proposed cut slope and which can be determined with site specific explorations. If right-of-way limits restrict the excavation of permanent cut slopes, retaining walls (described in Section 4.4) should be used to support cut slopes.

4.7 CULVERT DESIGN AND CONSTRUCTION CONSIDERATIONS

Presently, there are three large-diameter culverts which cross the project alignment between the City of Lynden and the Canadian border. Borings BH-7 through BH-9 were drilled near these culverts. As part of roadway widening, these culverts will need to be either replaced or extended. Depending on the proposed drainage plan, culvert extensions may be required as part of the project.

Based on the soil conditions encountered and the groundwater levels observed in our explorations, it is anticipated that groundwater may be encountered during installation of culverts, especially if the culverts are installed following an extended period of wet weather. If shallow groundwater is encountered during excavation of utility trenches, we expect that it can be controlled by using sumps and pumps; however, the need for and type of dewatering system required will depend on the final elevations of the culvert, and the groundwater level at the time of construction. Regardless of the dewatering system used, it should be installed and operated such that natural soils are prevented from being removed along with the groundwater.

Reasonable care should be taken to prevent groundwater from flowing in from the bottom of the excavation, thereby creating a “quick” condition. Under quick conditions, the density of the natural soils will be reduced, resulting in increased pipe settlement during and after installation of the culvert. To reduce the risk of creating a quick condition, we recommend the groundwater level be kept at least 0.6 meters (2 feet) below the bottom of the excavation.

During culvert installation, loose, organic soils encountered at/below the design subgrade elevation of the culvert should be overexcavated and replaced with compacted gravel fill. Overexcavation of these compressible soils should be performed to minimize differential settlement of the culvert and the overlying pavement section.

New culverts and culvert extensions should constructed as described in Section 7-02 and 7-08 of the 1998 WSDOT *Standard Specifications.* Bedding should consist of Gravel Backfill for Pipe Zone Bedding (Section 9-03.12(3)). Bedding should be placed in 6-inch lifts and compacted to 90 percent of the maximum dry density, as described in Section 7-08.3(1). Backfill should meet the general gradation requirements for Bank Run Gravel for Trench Backfill (9-03.19). Native soils may be used as pipe trench backfill provided the materials meet WSDOT specifications, or meet the approval of the engineer. Backfill should be placed and compacted as described as in Section 7-08.3(3).

4.8 DRAINAGE AND EROSION CONTROL MEASURES

Temporary and permanent drainage and erosion measures should be implemented during construction. Under no circumstances should water be allowed to pond adjacent to paved areas. The roadway, shoulder, and adjacent slopes should be graded and maintained such that surface drainage is directed away from the road into swales or other controlled drainage devices. All pavement drainage should be directed into conduits which carry runoff away from the new roadway, into a storm drain system or other appropriate outlet.

Erosion control measures should include limiting clearing and grading activities to minimize exposed areas, using straw mulch and erosion control matting to stabilize graded areas, placing filter fences and constructing sedimentation ponds to control runoff, hydroseeding areas after site grading is complete, and performing earthwork activities in the drier summer and early fall months. Implementation of these measures will help reduce rain and runoff impacts to exposed slopes.

5.0 CONTINUING GEOTECHNICAL ENGINEERING SERVICES

We recommend performing additional subsurface explorations, laboratory tests, and analysis to finalize the conclusions and preliminary recommendations presented herein. Specifically, these explorations will be focused on proposed bridge foundations and the new/widened causeway. We recommend additional explorations be performed at the following locations:

- At all bridge abutment or pier locations not explored as part of this study.
- Along the edge of the Wiser Lake causeway for the purposes of estimating sheet pile lengths and embedment depths.
- Along the Nooksack River bridges and approach embankments, for the purpose of examining the limits of peat and other compressible soils in this area and for determining preloading and surcharging recommendations.
- Along proposed cut and fill slopes where retaining walls will be required.

Additional laboratory testing should be performed to more accurately estimate potential settlements of soft, compressible soils. Testing should include consolidation tests, moistures content tests and determination of unit weight. Resistivity and pH tests should be performed to determine the corrosion potential of soils anticipated to be in contact with sheet piling and metal culverts.

6.0 UNCERTAINTIES AND LIMITATIONS

We have prepared this design file report for WSDOT and Parsons Brinckerhoff for use in conceptual planning and design of this project. This report is issued with the understanding that further explorations and studies will be performed for the final design.

The conclusions and interpretations presented herein should not be construed as a warranty of the subsurface conditions. Experience has shown that subsurface soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations and may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, HWA should be notified for review of the recommendations of this report, and revision of such if necessary.

Within the limitations of scope, schedule and budget, HWA attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology in the area at the time the report was prepared. No warranty, express or implied, is made.

We appreciate this opportunity to be of service.

Sincerely,

HWA GEOSCIENCES, INC.

David L. Sowers, P.E.
Geotechnical Engineer

Ralph N. Boirum, P.E.
Vice President

DLS:RNB:dls

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APPENDIX A

FIELD INVESTIGATION

APPENDIX A

FIELD INVESTIGATION

The field investigation for the SR 539 improvements project was performed May 15 through May 21, 1997, and consisted of drilling and sampling 9 exploratory borings and 18 shallow hand borings. The machine-drilled borings were advanced to depths ranging from 8.8 to 27.9 m (29 to 91½ ft) below existing ground surface; the hand borings were advanced to depths ranging from 0.6 to 3.0 m (2 to 10 ft) below existing ground surface.

Geotechnical drilling for the borings was performed by Holocene Drilling of Pacific, Washington under subcontract to HWA, using a Mobile B-61 truck mounted drill rig. The borings were advanced using 21 cm (8¼ in.) OD continuous flight, hollow stem augers.

At each boring location, sampling was performed in general accordance with ASTM D 1586 using a 50 mm (2 in.) OD split-spoon sampler and a standard 64 kg (140 lb.) automatic hammer. During the test, a sample is obtained by driving the sampler 450 mm (18 in.)into the soil with the hammer free-falling 760 mm (30 in.) per stroke. The number of blows required for each 150 mm (6 in.) of penetration is recorded. The Standard Penetration Resistance ("N-value") of the soil is recorded as the number of blows required for the final 300 mm (12 in.) of penetration. If a total of 50 blows is recorded within a single 150 mm (6 in.) interval, the test is terminated, and the blow count is recorded as 50 blows for the depth of penetration. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils.

The hand borings were drilled by HWA personnel using an 81 mm (3.2 in.) OD hand auger. At select intervals, non-standard penetration test (NSPT) sampling was performed using a 50 mm (2 in.) OD split-spoon sampler and an 18 kg (40 lb.) hammer. During the test, a sample is obtained by driving the sampler 300 mm (12 in.) into the soil with the 18 kg hammer free-falling 600 mm (24 in.) per stroke. The number of blows required for each 150 mm (6 in.) of penetration is recorded; the results of the NSPT provide a qualitative measure of the relative density of granular soils and the relative consistency of cohesive soils.

The explorations were drilled under the full-time observation of a HWA engineer. Soil samples obtained from the borings were classified in the field and representative portions were placed in plastic bags. These soil samples were then taken to our Lynnwood, Washington laboratory for further examination and testing. Pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence was recorded. The stratigraphic contacts shown on the individual logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The soil and groundwater conditions depicted are only for the specific dates and locations reported, and therefore are not necessarily representative of other locations and times.

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

HWA personnel performed laboratory tests in general accordance with appropriate American Society for Testing and Materials (ASTM) test methods. Laboratory tests were conducted on selected samples to characterize certain engineering properties of the on-site soils. Laboratory tests, as described below, included determination of moisture content, grain size distribution, fines content, and Atterberg Limits. The test procedures and test results are briefly discussed below.

Moisture Content

Natural moisture contents of selected samples were determined in general accordance with ASTM D 2216. Results are plotted on the exploration logs in Appendix A.

Grain Size Distribution

Grain size distribution and fines content were determined for selected samples in general accordance with ASTM D 422 and ASTM D 1140, respectively. Results of grain size analyses are plotted on Figures B-1 through B-5.

Atterberg Limits

The liquid limit (LL), plastic limit (PL), and plasticity index (PI) were determined for selected samples in general accordance with ASTM D 4318. The test results are plotted on Figure B-6.

APPENDIX C

PREVIOUS INVESTIGATIONS

APPENDIX C

PREVIOUS INVESTIGATIONS

To assist in planning our field exploration, we researched records at WSDOT offices in Tumwater, Olympia, and Seattle, Washington. A summary of subsurface data obtained from the WSDOT files is presented in Table C-1.